



Application of Partially Prestressing in Crack Control of Reinforced Concrete Structures

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Learning Objectives

1. History of Partial Prestressing
2. Partial Prestressing Design Approach
3. Crack Control Using Partial Prestressing
4. Potential applications

Presentation Outline

1. Background
2. Introduction to Partial Prestressing
3. Design Approach
4. Example of a Pier Cap Design
5. Potential applications

Background

To find a solution for the following issues:

- Excessive camber for full prestressing
- Constructibility issue due to high density reinforcing bars in RC
- Structural crack width control at Service Limit State for certain environmental conditions or structural elements
- Excessive deflection of reinforced concrete structure

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Introduction to Partial Prestressing

What is Partial Prestressing?

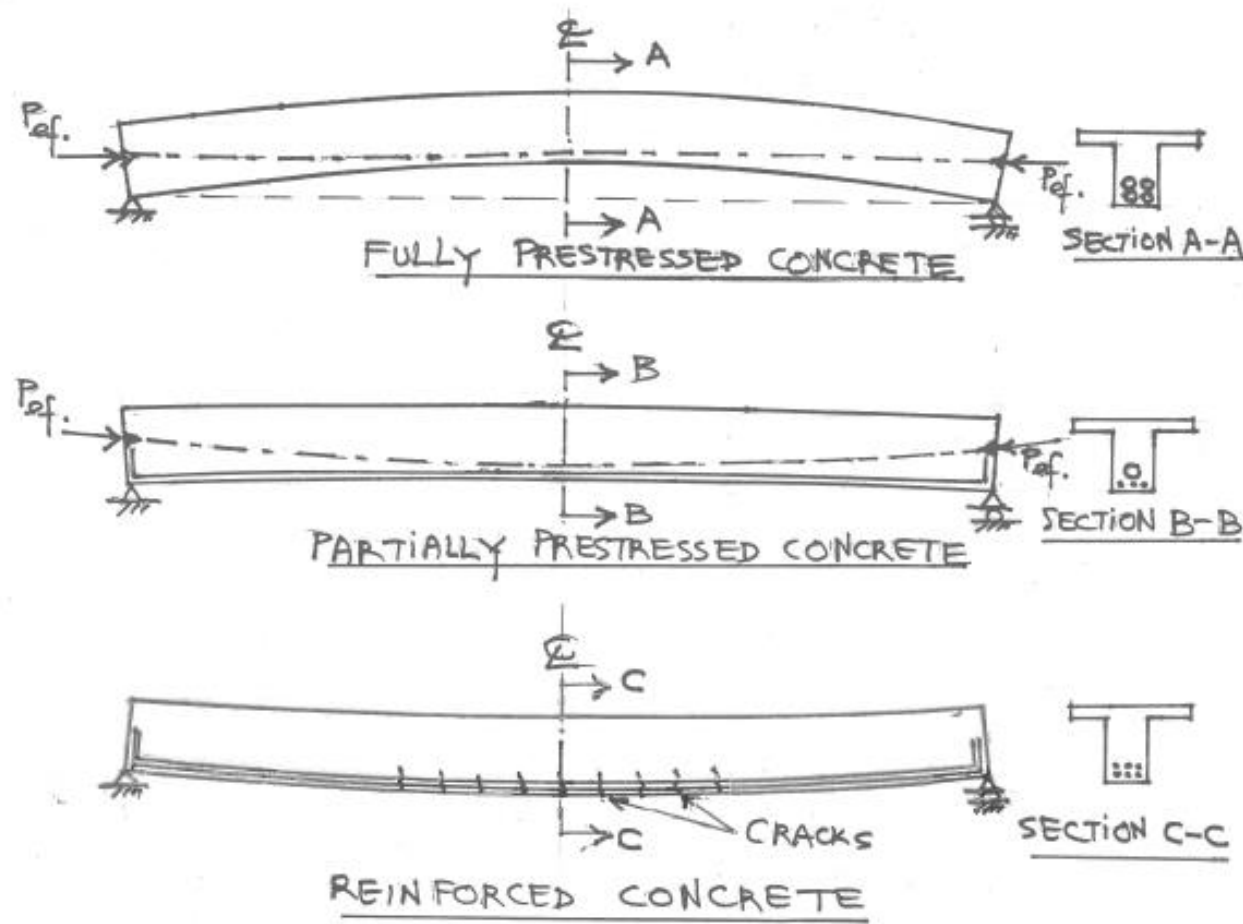
Partial prestressing is a structural concrete utilizing a combination of both prestressing Steel and passive reinforcing steel which allows tensile stresses and limited crack width at Service limit State load combinations and also satisfy Ultimate Limit Stage at the same time.

What is Full Prestressing?

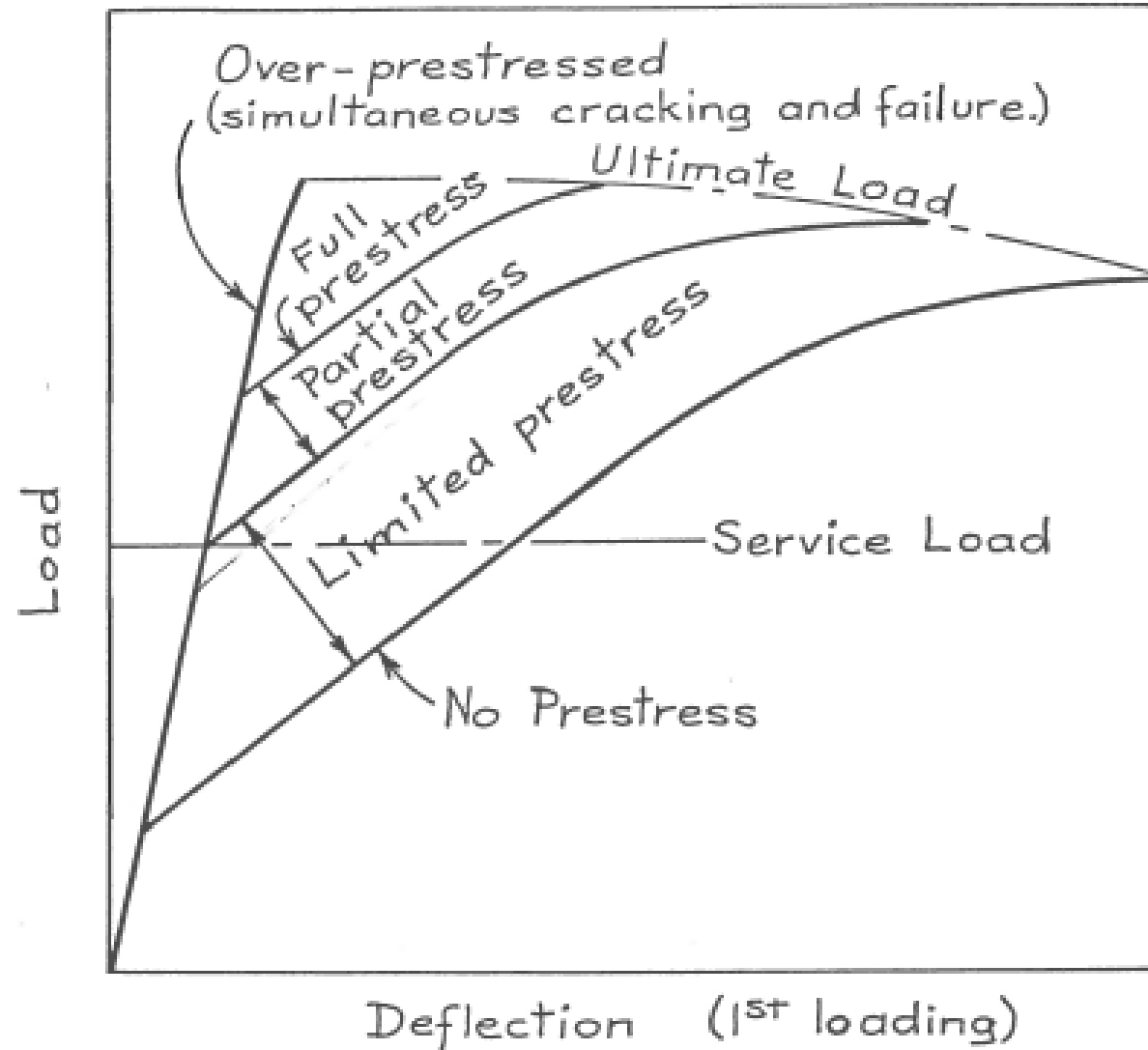
Full prestressing is a prestressed concrete with compression dominant and zero tension allowed at Service Limit Stage load combinations and also meet Ultimate Limit Stage.

The 2010 fib model Code is no longer differentiated between full prestressing, partial prestressing and Reinforced concrete. It treated the structural concrete as one continuous spectrum from reinforced Concrete to full prestressing. Some other countries in Europe, Switzerland and Australia design codes also adopting similar approach like fib.

Introduction to Partial Prestressing



Introduction to Partial Prestressing



Introduction to Partial Prestressing

Brief History of Partial Prestressing Concept

- 1930: Eugene Freyssinet of France was responsible for the development of full prestressing, used high strength steel to overcome concrete creep & shrinkage and established design criteria that the concrete is in compression (no tension is allowed). No ultimate strength check required.
- 1939: von Emperger of Austria recommended that ordinary reinforced concrete be provided with additional prestressing wires to control deflection and crack width, including checking of strength under ultimate load conditions.
- 1945: Abeles of UK suggested the non-prestressed reinforcement might consist of high strength steel of the same type would either be tensioned only a part of them to their full capacity, or all of them to an initial prestress well below that normally utilized in prestressing, including checking of strength under ultimate load condition. Abeles idea was opposed by Freyssinet who stated that “No half-way house” between prestressed and reinforced concrete.

Introduction to Partial Prestressing

The status of Partial Prestressing Concept Acceptance Around the World

- 1953: The West Germany Code of Practice for Prestressed Concrete (DIN 4227) introduced Limited (Partial) and full prestressing. The Code required minimum reinforcing steel of 0.3% of concrete cross section for Limited prestressing. No reinforcing bars required for full prestressing.
- 1959: British Code of Practice (CP115) accepted limited tension stress in prestressed concrete design.
- 1968: The Swiss Code SIA 162 adopted Partial Prestressing as official design practice in Switzerland. Currently, the SIA 162 adopted a unified approach to reinforced, partially prestressed and fully prestressed concrete. For railway bridges, no tension is allowed and fatigue must be checked.
- 1970: The Joint European Committee on Concrete (CEB-FIP), establishes three classes of prestressed concrete:
 - Class 1: Fully prestressed, no tensile stress is allowed at service load.
 - Class 2: Partially prestressed, occasional temporary cracking is allowed under infrequent high loads.
 - Class 3: Partially prestressed, permanent cracks with limited crack width is allowed under service loads.
- 1972: British Code of Practice (CP110) introduced Class 3 concrete which allowed cracks to be present under Service Loads (Partial Prestressing)
- 1978: Australian Code AS 1481 contains amendments related to design of partial prestressing.
- 2010: fib Model Code for Concrete Structures 2010 also adopted a unified approach to reinforced, partially prestressed, and fully prestressed concrete.

Introduction to Partial Prestressing

Advantages of Partial Prestressing Concept

1. Less prestressing steel (saving project budget).
2. Reduce camber in case of full prestressing.
3. Better control of crack width at service loads in case of reinforced concrete.
4. Better control of deflection at service loads in case of reinforced concrete.
5. Improvement in constructability in case of reinforced concrete.
6. Improvement in durability especially in extremely corrosive and high relative humidity
7. A better solution for cases where live loads is larger than dead loads.

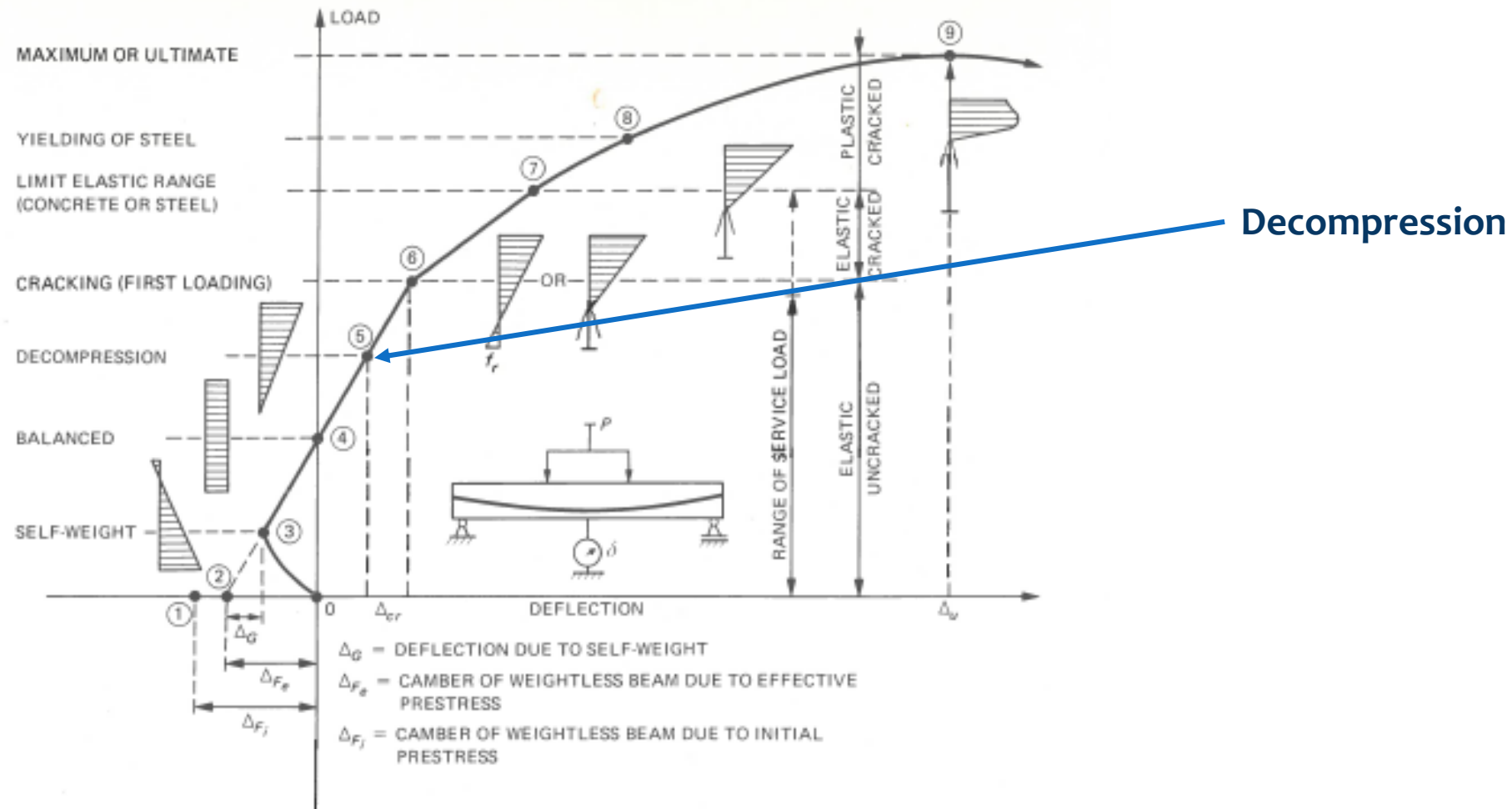
Disadvantage of Partial Prestressing Concept

1. Fatigue strength issue for repetitive live loads, e.g. train loads.
2. The accuracy in computing crack width at service limit state.

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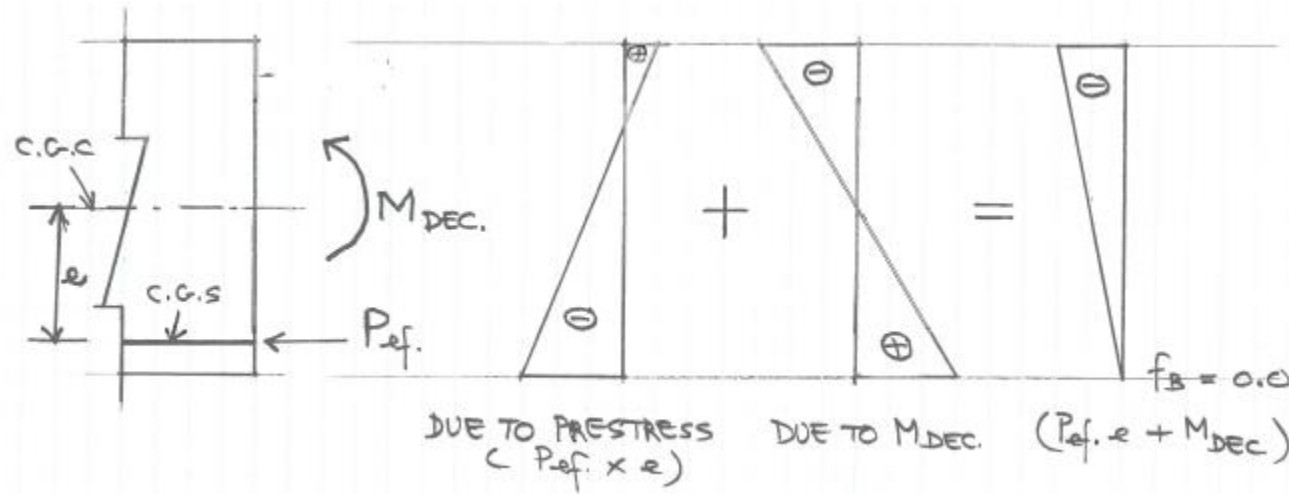
Design Approach



Load vs Deflection Curve (A.E. Naaman)

Design Approach

Statically Determinate Structures (Hugo Bahmann of Switzerland)

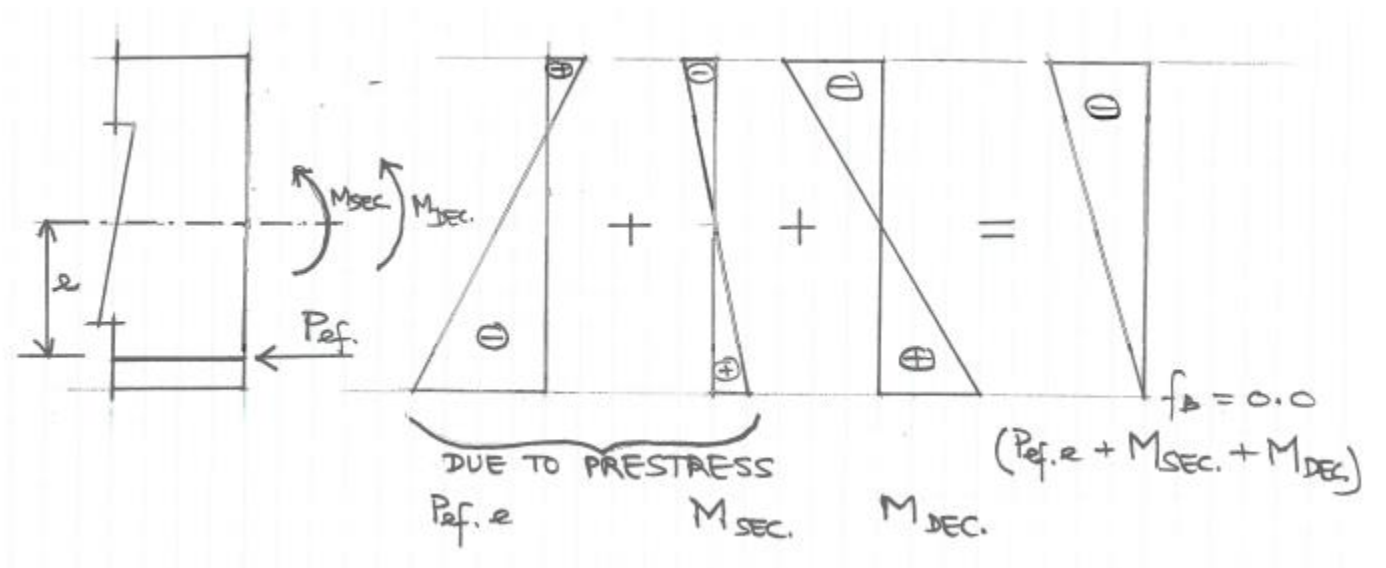


$M(Dec.)$: Decompression moment

The applied bending moment in combination with effective prestressing after all losses resulted in zero stress at the extreme fiber at which tensile stresses are caused by applied loads.

Design Approach

Statically Indeterminate Structures (Hugo Bahmann of Switzerland)



$M_{(Sec.)}$: Secondary moment due to prestressed after all losses

Design Approach

Degree of Prestress (Hugo Bachman of Switzerland / SIA 162)

1. Service Load Degree of Prestress

$$DP1 = \frac{M_{DEC.}}{M_{(D+L)}}$$

2. Permanent Load Degree of Prestress

$$DP2 = \frac{M_{DEC.}}{M_D}$$

Notes:

DP1 = 0.0 (No prestressing)

DP1 = 1.0 (Full prestressing)

DP2 = 1.0 (Full prestressing for permanent loads only)

Design Approach

Prestressing Index (A.E. Naaman / AASHTO LRFD)

$$PPR = \frac{A_{ps} \cdot f_{py}}{A_{ps} \cdot f_{py} + A_s \cdot f_y}$$

Where:

PPR = Partial Prestressing Ratio

Notes:

- PPR = 0.0 (No prestressing)
- AASHTO LRFD is no longer included PPR in the current edition

Design Approach

Step by Step Design Procedures

1. Select bending moment to be supported by PT (Decompression bending moment)
2. Determine the PT forces required
3. Determine the area reinforcing steel (non PT)
4. Detailing
5. Compute crack width
6. Compute deflection
7. Compute flexural strength of combined of PT and reinforcing bars

Notes: Iteration may be necessary in order to obtain the suitable PT and reinforcing bars to meet both Serviceability and ultimate limit states.

Design Approach

Select bending moment to be supported by PT

$$DP2 \geq 1.0 \longrightarrow M(\text{Dec.}) \geq 1.0 M(D)$$

DP2 < 1.0 should be considered for cases with live loads are much smaller than the permanent loads

Consideration for selecting DP2

1. Durability, environment, crack width limitation
2. Economic
3. Constructibility / detailing
4. Deformation
5. Fatigue, e.g. structure with repetitive live loads such as train (select DP1 = 1.0)

Design Approach

Determine PT forces required

$$P_{ef} = \frac{M_{DEC.} + M_{SEC.}}{e + k_1}$$

$$P_i = \frac{P_{ef.}}{\eta}$$

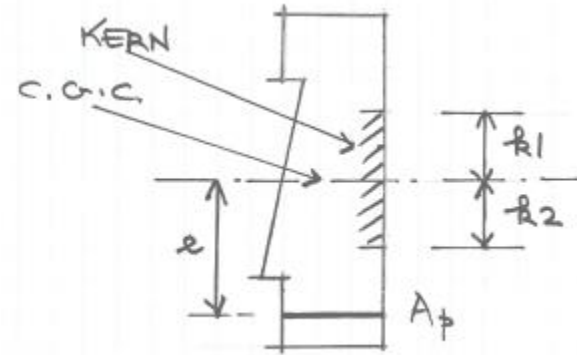
Where:

e = PT tendon eccentricity

k_1 = distance from centroid of uncracked section to kern limit opposite to center of gravity of PT tendon, e.g. if the PT is below the concrete centroid, k_1 is the top kern limit.

P_i = Initial prestressing force prior to longterm loss of prestress.

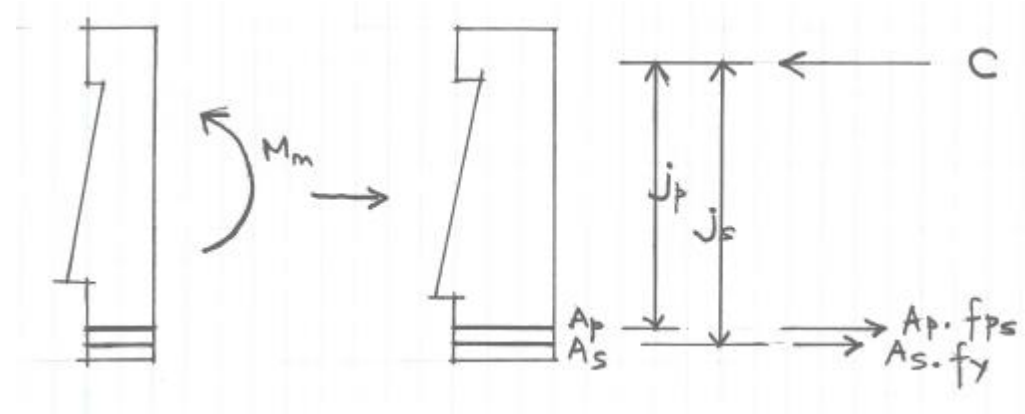
η = Approximate ratio of $P_{ef.}$ over P_i ranges from 0.85 to 0.9



Design Approach

Determine the area of reinforcing steel

$$M_u \leq \phi M_m$$
$$M_m = A_p \cdot f_{ps} \cdot j_p + A_s \cdot f_y \cdot j_s$$
$$A_s = \frac{M_m - A_p \cdot f_{ps} \cdot j_p}{f_y \cdot j_s}$$



Where:

M_u = Factored ultimate moment of a section

M_n = Nominal flexural resistance

Φ = Resistance factor per LRFD Article 5.5.4.2

f_{ps} = Average prestressing steel stress at nominal resistance

Min. reinforcing steel recommended is 0.3 % – 0.8 % of area of concrete tension zone.

Design Approach

Detailing

Consider the following factors in PT and reinforcing bars detailing:

1. Consider smaller spacing of reinforcing bars so that the cracks width is limited and well distributed.
2. Place the reinforcing bars as close as possible to the extreme tension face
3. PT tendons should be confined by the reinforcing bars and apply practical standard practice in deciding tendon size.
4. Apply common sense to balance the number of reinforcing bars vs tendons to avoid constructability Problem at critical locations. If necessary increase the degree of prestress and reduce reinforcing bars.

Design Approach

Cracks Control

- Cracks are unavoidable in structural concrete
- Cracks and crack width can be control by design

Reasons for Crack Control

- Appearance, aesthetic, public concern
- Risk of reinforcement corrosion in aggressive environment
- Risk of water and gas intrusion
- Cracks can change the structural behavior such as unexpected deformations / deflections

Advantages

- Cracks in concrete confirm the behavior of the structural concrete
- For seismic resistance structures, cracks could dampening seismic forces
- Cracks can dampening vehicle and ship impact forces

Design Approach

Crack width Limit

FDOT: Standard Specification Section 400-21 Disposition of Cracked Concrete

Table 2 DISPOSITION OF CRACKED CONCRETE BRIDGE DECKS [see separate Key of Abbreviations and Footnotes for Tables 1 and 2]													
Elev. Range	Crack Width Range (inch) ⁽²⁾ x = crack width	Cracking Significance Range per LOT ⁽¹⁾											
		Isolated less than 0.005%				Occasional 0.005% to<0.017%			Moderate 0.017% to<0.029%			Severe 0.029% or gtr.	
		Environment Category											
		S A	MA	EA	SA	M A	EA	SA	MA	EA	S A	M A	E A
Elevation: 12 feet or Less AMHW	x ≤ 0.004	N T	NT	NT	NT	NT	NT	NT	NT	NT			
	0.004< x ≤ 0.008	N T	NT	EI/ M	NT	NT	EI/M	EI/M	EI/ M	EI/M			
	0.008< x ≤ 0.012	N T	NT	EI/ M	NT	EI/ M	EI/M	EI/M	EI/ M				
	0.012< x ≤ 0.016	N T	NT	EI/ M	NT	EI/ M							
	0.016< x ≤ 0.020	EI /M	EI/ M	EI	EI								
	0.020< x ≤ 0.024	EI /M	EI	EI		Investigate to Determine Appropriate Repair ^(4, 5) or Rejection					Reject and Replace		
	0.024< x ≤ 0.028	EI /M	EI										
	x > 0.028												

$0.004'' = 0.1 \text{ mm}$

$0.008'' = 0.2 \text{ mm}$

$0.012'' = 0.3 \text{ mm}$

Elevation: Over Land or More Than 12 feet AMHW	Crack Width	S A	MA	EA	SA	M A	EA	SA	MA	EA	S A	M A	E A
	$x \leq 0.004$	N T	NT	NT	NT	NT	NT	NT	NT	NT			
	$0.004 < x \leq 0.008$	N T	NT	NT	NT	NT	EI/M	NT	EI/ M	EI/M			
	$0.008 < x \leq 0.012$	N T	NT	EI/ M	NT	NT	EI/M	EI/M	EI/ M				
	$0.012 < x \leq 0.016$	N T	NT	EI/ M	NT	EI/ M							
	$0.016 < x \leq 0.020$	N T	EI/ M	EI	EI/ M		Investigate to Determine Appropriate Repair ^(4,5) or Rejection						
	$0.020 < x \leq 0.024$	N T	EI/ M	EI							Reject and Replace		
	$0.024 < x \leq 0.028$	N T	EI/ M										
	$x > 0.028$												

Design Approach

Crack width Limit

FDOT: Standard Specification Section 400-21 Disposition of Cracked Concrete



Key of Abbreviations and Footnotes for Tables 1 and 2		
Type Abbreviation	Abbreviation	Definition
Repair Method	EI	Epoxy Injection
	M	Methacrylate
	NT	No Treatment Required
	PS	Penetrant Sealer
Environment Category	EA	Extremely Aggressive
	MA	Moderately Aggressive
	SA	Slightly Aggressive
Reference Elevation	AMHW	Above Mean High Water
Footnotes		
(1) Cracking Significance Range is determined by computing the ratio of Total Cracked Surface Area (TCSA) to Total Surface Area (TSA) per LOT in percent $[(TCSA/TSA) \times 100]$ then by identifying the Cracking Significance Range in which that value falls. TCSA is the sum of the surface areas of the individual cracks in the LOT. The surface area of an individual crack is determined by taking width measurements of the crack at 3 representative locations and then computing their average which is then multiplied by the crack length.		
(2) Crack Width Range is determined by computing the width of an individual crack as computed in (1) above and then identifying the range in which that individual crack width falls.		
(3) When the Engineer determines that a crack in the 0.004 inch to 0.008 inch width range cannot be injected then for Table 1 use penetrant sealer unless the surface is horizontal, in which case, use methacrylate if the manufacturer's recommendations allow it to be used and if it can be applied effectively as determined by the Engineer.		
(4) (a) Perform epoxy injection of cracks in accordance with Section 411. Seal cracks with penetrant sealer or methacrylate as per Section 413. (b) Use only methacrylate or penetrant sealer that is compatible, according to manufacturer's recommendations, with previously applied materials such as curing compound or paint or remove such materials prior to application.		
(5) When possible, prior to final acceptance of the project, seal cracks only after it has been determined that no additional growth will occur.		
(6) Methacrylate shall be used on horizontal surfaces in lieu of penetrant sealer if the manufacturer's recommendations allow it to be used and if it can be applied effectively as determined by the Engineer.		
(7) Unless directed otherwise by the Engineer, repair cracks in bridge decks only after the grinding and grooving required by 400-15.2.5 is fully complete.		

Design Approach

Crack width Limit

Fib Model Code for Concrete Structure 2010

	RC	PL1	PL2	PL3
XO	0.3	0.2	0.3	0.3
XC	0.3	0.2	0.3	0.3
XD	0.2		0.2	0.2
XS	0.20		0.2	0.2
XF	0.2		0.2	0.2

0.2 mm = 0.008"
0.3 mm = 0.012"

Table 7.6-1: Crack width limit (mm) for reinforced members and prestressed members with bonded prestressing

XO: No risk of corrosion, e.g. very dry environment

XC: Corrosion induced by carbonation

XD: Corrosion induced by chloride other than sea water

XS: Corrosion induced by chloride from sea water

XF: Freezing and thawing attack

Florida is considered in PL2 category

Design Approach

Crack width Limit by Distribution Of Reinforcement (AASHTO LFD)

8.16.8.4 Distribution of Flexural Reinforcement

To control flexural cracking of the concrete, tension reinforcement shall be well distributed within maximum flexural zones. When the design yield strength, f_y , for tension reinforcement exceeds 40,000 psi, the bar sizes and spacing at maximum positive and negative moment sections shall be chosen so that the calculated stress in the reinforcement at service load f_s in ksi does not exceed the value computed by:

$$f_s = \frac{z}{(d_c A)^{1/3}} \leq 0.6 f_y \quad (8-61)$$

where:

8.16.8.4

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A = effective tension area, in square inches, of concrete surrounding the flexural tension reinforcement and having the same centroid as that reinforcement, divided by the number of bars or wires. When the flexural reinforcement consists of several bar or wire sizes, the number of bars or wires shall be computed as the total area of reinforcement divided by the area of the largest bar or wire used. For calculation purposes, the thickness of clear concrete cover used to compute A shall not be taken greater than 2 in.

d_c = distance measured from extreme tension fiber to center of the closest bar or wire in inches. For

calculation purposes, the thickness of clear concrete cover used to compute d_c shall not be taken greater than 2 inches.

The quantity z in Equation (8-61) shall not exceed 170 kips per inch for members in moderate exposure conditions and 130 kips per inch for members in severe exposure conditions. Where members are exposed to very aggressive exposure or corrosive environments, such as deicer chemicals, protection should be provided by increasing the denseness or imperviousness to water or furnishing other protection such as a waterproofing protecting system, in addition to satisfying Equation (8-61).

Design Approach

Crack width Limit by Distribution Of Reinforcement (AASHTO LRFD)

The spacing s of mild steel reinforcement in the layer closest to the tension face shall satisfy the following:

$$s \leq \frac{700\gamma_e}{\beta_s f_{ss}} - 2d_c \quad (5.7.3.4-1)$$

in which:

$$\beta_s = 1 + \frac{d_c}{0.7(h - d_c)}$$

where:

- γ_e = exposure factor
 - = 1.00 for Class 1 exposure condition
 - = 0.75 for Class 2 exposure condition
- d_c = thickness of concrete cover measured from extreme tension fiber to center of the flexural reinforcement located closest thereto (in.)
- f_{ss} = calculated tensile stress in mild steel reinforcement at the service limit state not to exceed $0.60 f_y$ (ksi)
- h = overall thickness or depth of the component (in.)

Design Approach

Crack width Limit

ACI Committee 224

Exposure Condition	Crack Width (in.)	Crack Width (mm)
Dry air or protective membrane	0.016	0.41
Humidity, moist air, soil	0.012	0.33
De-icing chemical	0.007	0.18
Sea water and sea water spray; wetting and drying	0.006	0.15
Water retaining structures	0.004	0.10

Design Approach

Crack width limit

CP 110

Table 6.1 Hypothetical flexural tensile stresses - partially prestressed members
(Table 34 in CP 110)

	Limiting Nominal Crack Width (mm)	Allowable Tensile Stress (MPa), for Concrete Strength Grade (MPa):		
		30	40	50
A Pretensioned tendons	0.1	-	4.1	4.8
	0.2	-	5.0	5.8
B Grouted post-tensioned tendons	0.1	3.2	4.1	4.8
	0.2	3.8	5.0	5.8
C Pretensioned tendons distributed in the tensile zone and positioned close to the tension faces of the concrete	0.1	-	5.3	6.3
	0.2	-	6.3	7.3

Table 6.2 Depth factors for tensile stresses - partially prestressed members
(Table 35 in CP 110)

Depth of Member (mm)	200 (and under)	400	600	800	1000 (and over)
Factor	1.1	1.0	0.9	0.8	0.7

Design Approach

Crack Width Determination

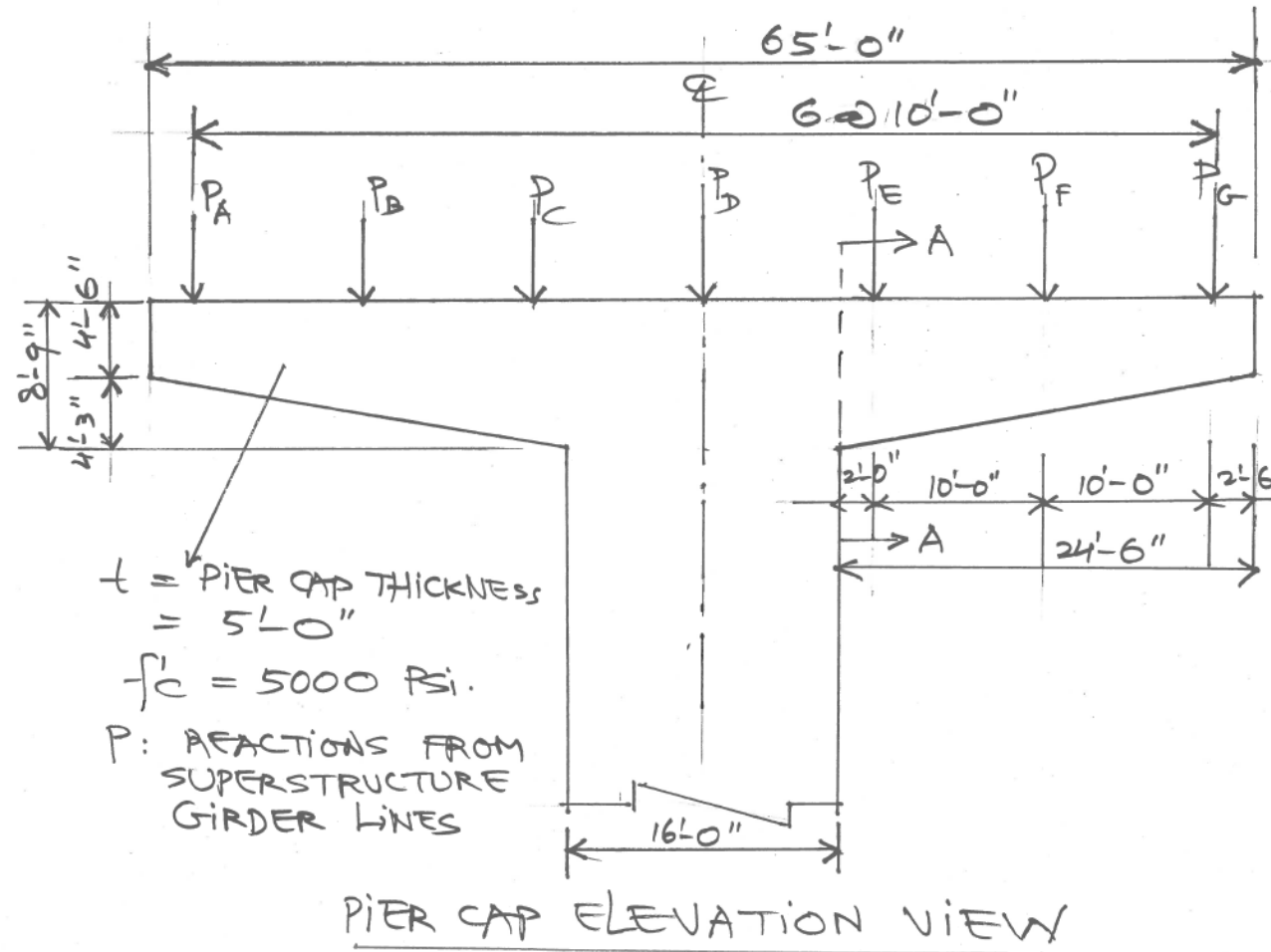
There are several methods in predicting crack width for prestressed concrete:

1. Method 1: Based on hypothetical tensile concrete stresses at extreme fiber in an un-cracked section (simple , not accurate, for bonded PT only), e.g. CP-110
2. Method 2: Based on steel average steel stress / strain and crack spacing computed by cracked section analysis, e.g. fib model Code.
3. Method 3: Based on steel stress at the crack, concrete cover, and area of concrete around each bar, Gergely-Lutz equation adopted by ACI and AASHTO

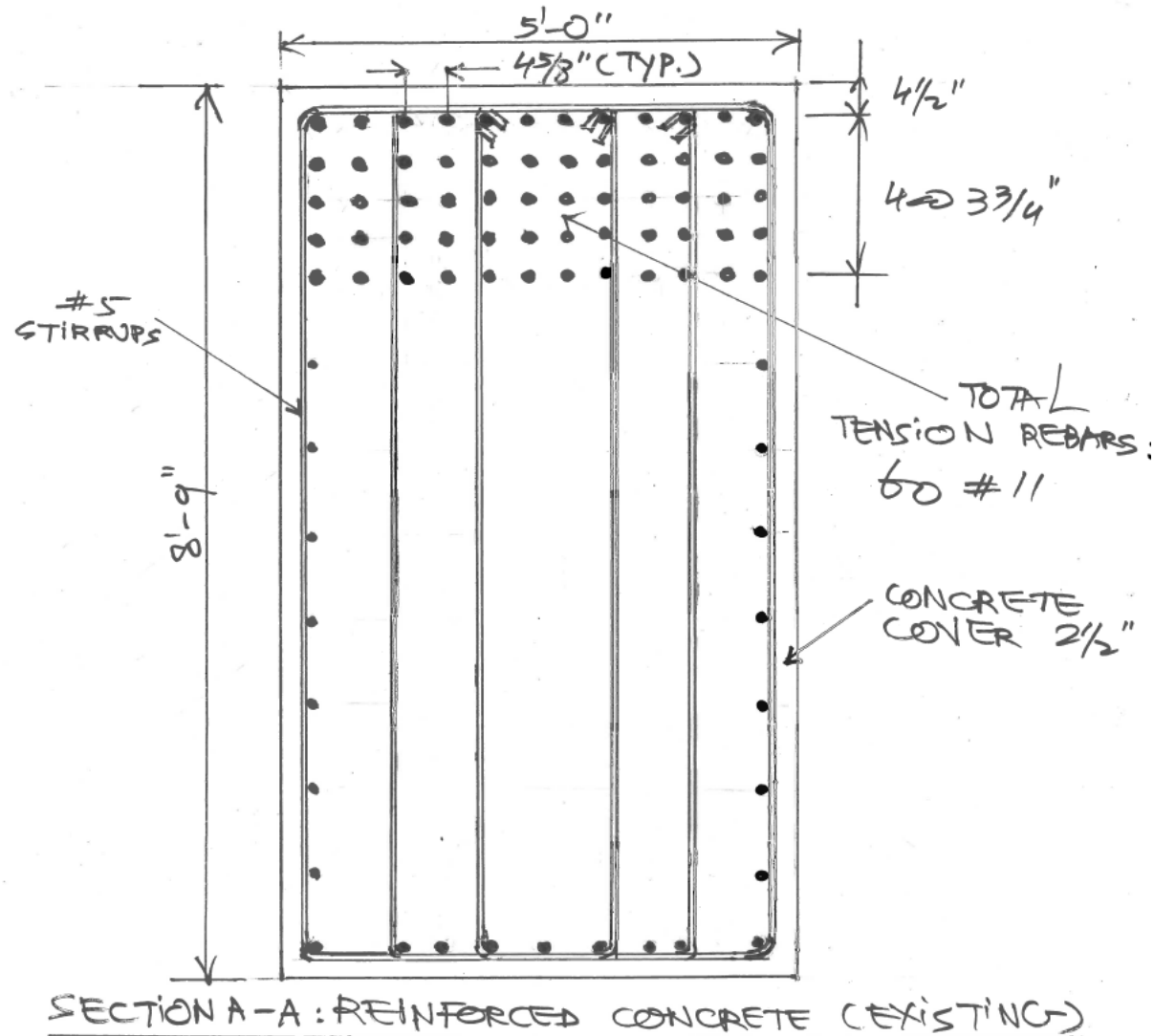
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Example of a Pier Cap Design



Example of a Pier Cap Design



Designed by:
AASHTO LFD Design Spec

Example of a Pier Cap Design

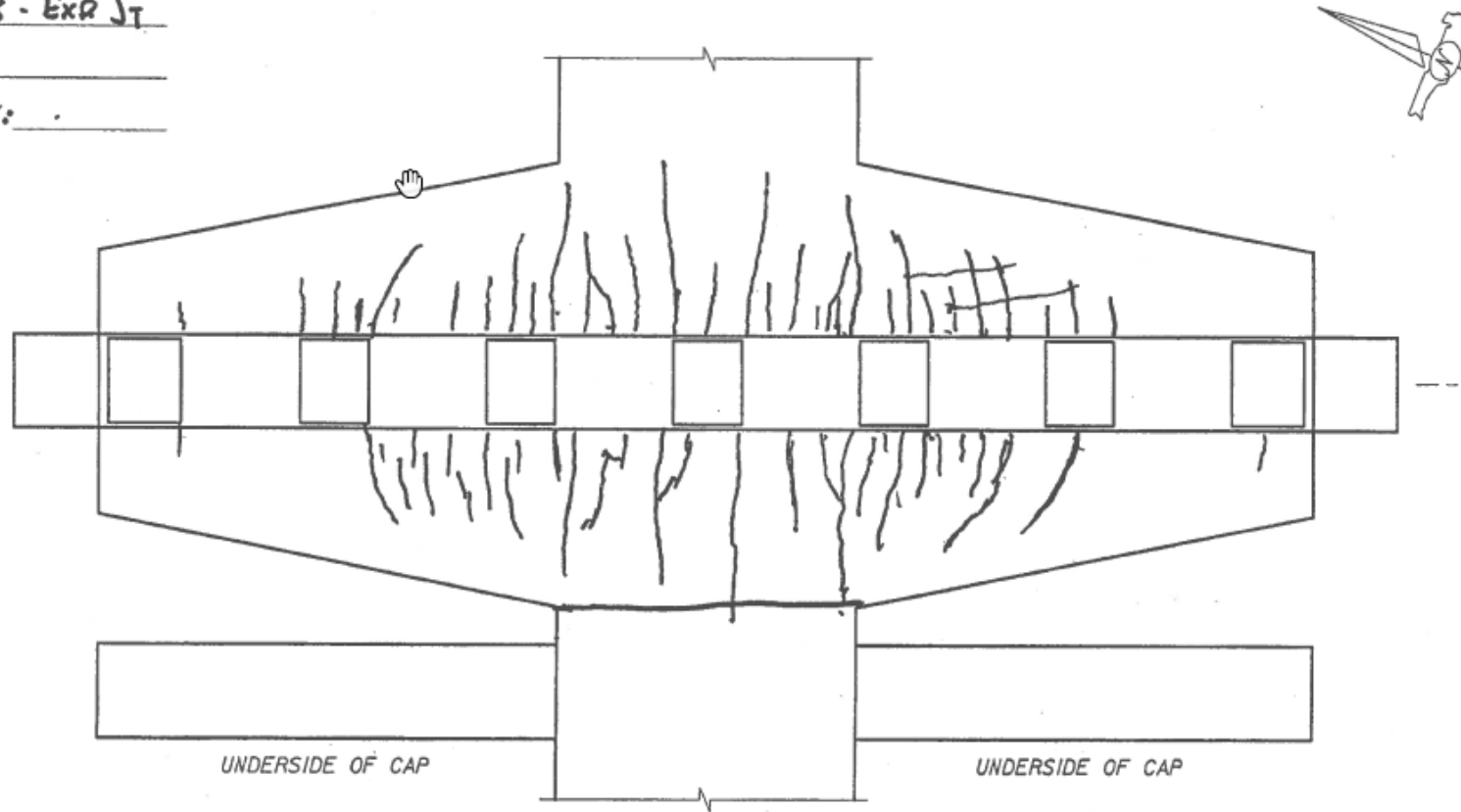


Example of a Pier Cap Design

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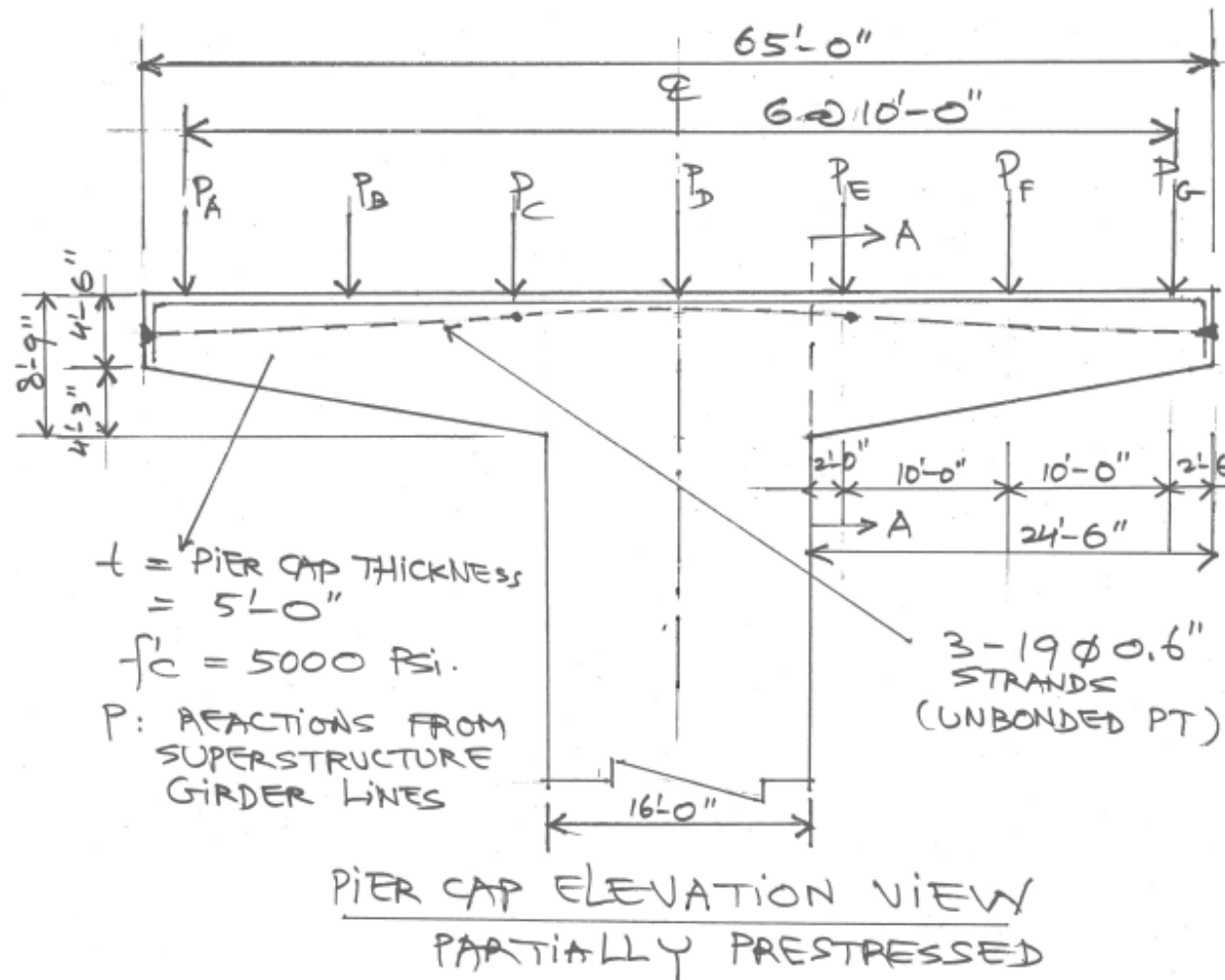
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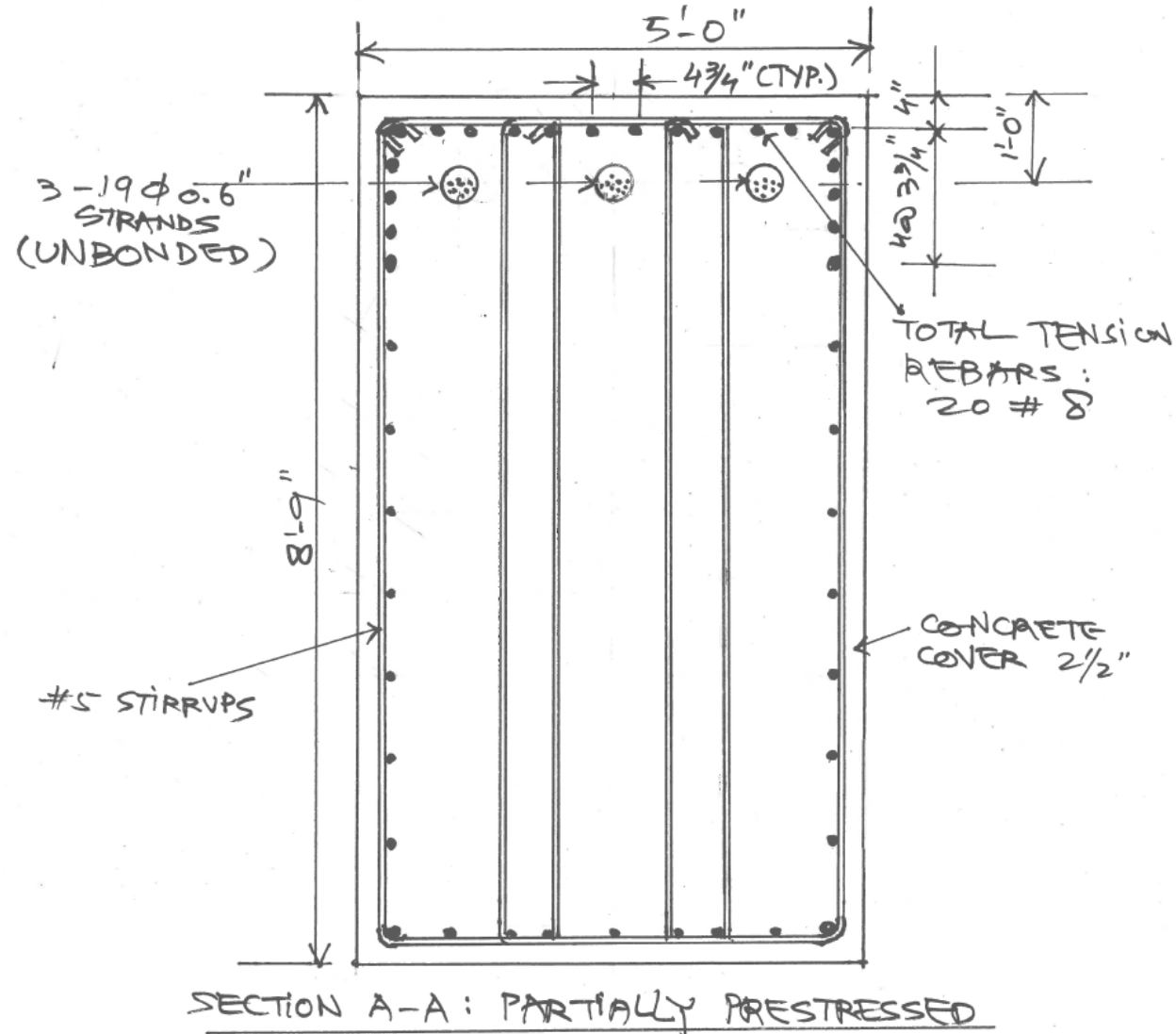
Typical cracks pattern

Measured crack width varies from 0.005" to 0.013"

Example of a Pier Cap Design




Example of a Pier Cap Design



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Example of a Pier Cap Design

 Structures Design Office	FPID No.	Sheet No. 1 of
	Bridge	Originator: TT Date: 6/1/17
	Subject	Reviewer: Date:

OBJECTIVE: Redesign R.C. pier cap with partially prestressed concrete

SPECIFICATIONS: Since the original design used AASHTO LFD, the redesigned will adopt AASHTO LFD also.

Given:


Concrete strength $f'_c = 5000$ psi
 Unit weight = 145 lbs/cf.
 Ordinary reinforcing bars:
 Grade 60. (deformed)

New design:

$f'_c = 5000$ psi.
 Reinforcing bars: Grade 60 (deformed)

PT strands: 0.6" - grade 270 - low relaxation.

PT tendons are unbonded (flexible filler).

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SUMMARY OF BENDING MOMENT.

$$M_{DL1} = -1287.41 \text{ K-FT (PIER CAP)}$$

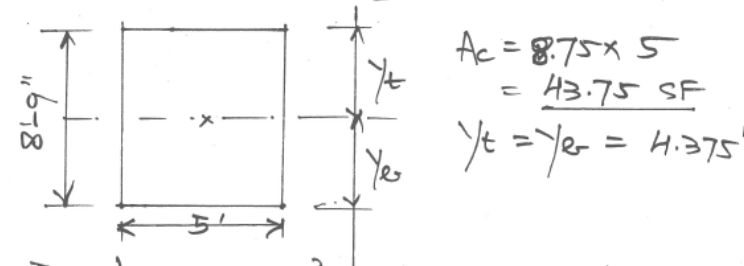
$$M_{DL2} = -11,908.00 \text{ K-FT (Girders)}$$

$$\Sigma M_{DL} = -13,195.41 \text{ K-FT}$$

$$M_{(L+I)} = -4198.00 \text{ K-FT}$$

$$M_{DL} + M_{(L+I)} = -17,393.41 \text{ K-FT}$$

SECTION PROPERTIES



$$I = \frac{1}{12} \times 5 \times 8.75^3 = 279.13 \text{ FT}^4$$


$$S_t = \frac{I}{y_t} = 279.13 / 4.375 = 63.80 \text{ FT}^3$$

$$S_b = \frac{I}{y_b} = 63.80 \text{ FT}^3$$

$$K_e = K_t = S_b / A_c = 63.8 / 43.75 = 1.458'$$

$$i^2 = \frac{I}{A_c} = 279.13 / 43.75 = 6.38 \text{ FT}^2$$

Example of a Pier Cap Design

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	Bridge	PIER CAP	Originator: TT Date: 6/1/17
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DECOMPRESSION

Selct, $DP2 = 1.0$

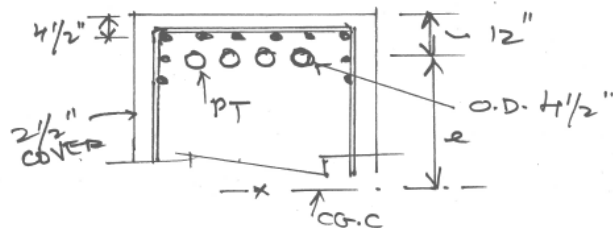
$$DP2 = M_{DEC.} / M_{DL} = 1.0$$

$$M_{DEC.} = 13,195.41 \text{ K-FT.}$$

$$DP1 = \frac{M_{DEC.}}{M_{CDL+LL+I}}$$

$$= 13,195.41 / 17,393.41$$

$$= 0.75$$



$$e = 4.375' - 1.0' = 3.375'$$


$$P_{ef} = \frac{M_{DEC.} + M_{SEC.}}{(D + KI)}$$

$$= \frac{13,195.41}{(3.375 + 1.458)} = 2730.27 \text{ Kips}$$

Assume 60% UTS after all losses

Number of 0.6" strand required:

$$= 2730.27 / 0.6 \times 57.6 = 77.6$$

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	Bridge	PIER CAP	Originator: TT Date: 6/1/17
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Use: 4-19 ϕ 0.6" PT

check stresses at initial

$$P_i = \frac{P_{ef}}{\eta} = \frac{2730.27}{0.85} = 3212.00 \text{ Kips}$$

Pier cap DL only.

$$f_t = -\frac{P_i}{A_c} - \frac{P_i \cdot e}{S_t} + \frac{M_{CDL}}{S_t}$$

$$= -\frac{3212.00}{43.75} - \frac{(3212.00)(3.375)}{63.8} + \frac{1287.41}{63.8}$$

$$= -73.42 - 169.91 + 20.18 = -223.15 \text{ KSF}$$

$$= -1549.65 \text{ Psi}$$

O.K.

$$f_c = -73.42 + 169.91 - 20.18$$

$$= +76.31 \text{ KSF}$$


$$= +530 \text{ Psi}$$

$$> 7.5 \sqrt{f'_c} = 7.5 \sqrt{0.7 \times 5000}$$

$$= 443.7 \text{ Psi.}$$

O.K.
Minor cracks.

Example of a Pier Cap Design

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Check stresses at service loads.

$$P_{ef} = 2730.27 \text{ Kips.}$$

$$M_{DL+LL+E} = -17,393.41 \text{ K-FT.}$$

$$f_t = -\frac{P_{ef}}{A_c} - \frac{P_{ef} \cdot e}{S_t} + \frac{M_{DL+LL+E}}{S_t}$$

$$= -\frac{2730.27}{43.75} - \frac{(2730.27)(3.375)}{63.8} + \frac{17,393.41}{63.8}$$

$$= -62.41 - 144.43 + 272.62$$

$$= +65.78 \text{ KSF}$$

$$= +456.8 \text{ Psi} < \frac{7.5 \sqrt{f'_c}}{7.5 \sqrt{5000}} = 530.33 \text{ Psi.}$$

Not crack yet!


$$f_c = -62.41 + 144.43 - 272.62$$

$$= -190.60 \text{ KSF}$$

$$= -1323.61 \text{ Psi} < -0.4 f'_c = -2000 \text{ Psi.}$$

O.K.

Try to lower the degree of prestress (DP2)

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$$\begin{aligned} \text{TRY } DP2 &= 0.75 \\ \therefore M_{DE} &= -0.75 (13,195.41) \\ &= -9896.55 \text{ K-FT.} \end{aligned}$$

$$\therefore DP1 = \frac{9896.55}{17,393.41} = 0.569$$

$$P_{ef} = \frac{9896.55}{(3.375 + 1.458)} = 2047.70 \text{ Kips}$$

$$\begin{aligned} \text{Number of 0.6" strand required:} \\ \frac{2047.70}{0.6 \times 58.6} &= 58 \end{aligned}$$

$$\text{Use: } 3-19 \phi 0.6"$$

$$\therefore P_{ef} = 3 \times 19 \times 58.6 \times 0.6 = 2004 \text{ Kips}$$

$$\therefore P_i = 2004 / 0.85 = 2357.6 \text{ Kips.}$$

Check concrete stresses at initial

$$f_t = -\frac{P_i}{A_c} - \frac{P_i \cdot e}{S_t} + \frac{M_{CDL}}{S_t}$$


$$= -\frac{2357.6}{43.75} - \frac{2357.6(3.375)}{63.8} + \frac{1287.41}{63.8}$$

$$= -53.88 - 124.72 + 20.18$$

$$= -158.42 \text{ KSF}$$

$$= -1100 \text{ Psi} < -0.55 f_{ci} = -1925 \text{ Psi. O.K.}$$

Example of a Pier Cap Design

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$$\begin{aligned}
 f_g &= -53.88 + 124.72 - 20.18 \\
 &= +50.66 \text{ KSF} \\
 &= +351.80 < 7.5 \sqrt{f'_c} \\
 &= 7.5 \sqrt{6.7 \times 5000} \\
 &= 443.7 \text{ PSI} \\
 &\text{O.K.}
 \end{aligned}$$

check concrete stresses at service loads


$$P_{ef} = 2004 \text{ Kips}$$

$$M_{DL+(L+I)} = -17,393.41 \text{ K-FT.}$$

$$\begin{aligned}
 f_t &= -\frac{P_{ef}}{A_c} - \frac{P_{ef} \cdot e}{S_t} + \frac{M_{DL+(L+I)}}{S_t} \\
 &= -\frac{2004}{43.75} - \frac{2004 \times 3.375}{63.80} + \frac{17,393.41}{63.80} \\
 &= -45.80 - 106.01 + 272.62 \\
 &= +120.81 \text{ KSF} \\
 &= +839.00 \text{ PSI} < 11.86 \sqrt{f'_c} \\
 &> 7.5 \sqrt{5000} \\
 &= 530.33 \text{ PSI.}
 \end{aligned}$$

The concrete is cracked and need to check crack width.

$$\begin{aligned}
 f_k &= -45.80 + 106.01 - 272.62 \\
 &= -212.41 \text{ KSF} \\
 &= |-1475.00 \text{ PSI}| < \frac{1 - 0.4 f'_c}{1 - 2000 \text{ PSI}} \\
 &\text{O.K.}
 \end{aligned}$$

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check concrete stresses at service loads when girders are removed.


$$P_{ef} = 2004 \text{ Kips}$$

$$M_{CDL} = -1287.41 \text{ K-FT.}$$

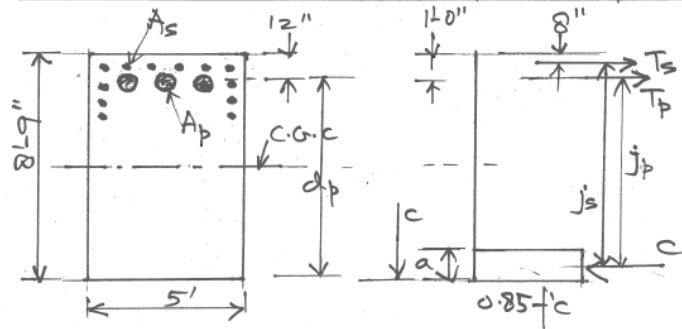
$$\begin{aligned}
 f_t &= -\frac{P_{ef}}{A_c} - \frac{P_{ef} \cdot e}{S_t} + \frac{M_{CDL}}{S_t} \\
 &= -\frac{2004}{43.75} - \frac{2004 \times 3.375}{63.80} + \frac{1287.41}{63.8} \\
 &= -45.80 - 106.01 + 20.18 \\
 &= -131.63 \text{ KSF} \\
 &= |-914.00 \text{ PSI}| < \frac{1 - 0.4 f'_c}{1 - 2000 \text{ PSI}} \\
 &\text{O.K.}
 \end{aligned}$$

$$\begin{aligned}
 f_g &= -45.80 + 106.01 - 20.18 \\
 &= +40.03 \text{ KSF} \\
 &= +277.98 \text{ PSI} < 7.5 \sqrt{5000} \\
 &= 530.33 \text{ PSI} \\
 &\text{O.K.}
 \end{aligned}$$

Example of a Pier Cap Design

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COMPUTE REINFORCING BARS REQUIRED.




$$\begin{aligned}
 M_u &= 1.3 M_{DL} + 1.67 M_{(L+L)} \\
 &= 1.3 (13,195.41) + 1.67 (4198) \\
 &= 24,165.00 \text{ K-FT. (C-)}
 \end{aligned}$$

compute unbonded PT stresses at ultimate.

$$f_{pu} = f_{pe} + 900 \left[\frac{(d_p - y_u)}{I_e} \right] \leq f_{py}$$

where:
 d_p = distance from the PT centroid to the extreme compressive fiber.
 y_u = distance from the extreme compressive fiber to centroid of neutral axis assuming PT tendons has yielded.

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$$\begin{aligned}
 \beta_1 &= 0.85 - \left(\frac{f'_c - 4000}{1000} \right) \times 0.05 \\
 &= 0.85 - \left(\frac{5000 - 4000}{1000} \right) \times 0.05 \\
 &= 0.8
 \end{aligned}$$

$$a = \beta_1 \times c$$

$$I_e = \frac{I_i}{(1 + 0.5 N_s)}$$

N_s = Number of support hinge between the two anchorages.
 $= 1.0$

$$I_e = \frac{65'}{(1 + 0.5 \times 1)} = 43'$$

Assume $f_{py} = 0.9 \times 270 = 243 \text{ KSI.}$

$$\begin{aligned}
 f_{py} &= 3 \times 19 \times 58.6 \times 0.9 \\
 &= 3006.18 \text{ Kips.}
 \end{aligned}$$

$$C = 0.85 f'_c \times a \times 60 = 3006.18 \times 10^3$$

$$a = \frac{3006.18 \times 10^3}{0.85 \times 5000 \times 60} = 11.78"$$

$$c = \frac{a}{\beta_1} = \frac{11.78}{0.8} = 14.73"$$

$$y_u = c = 14.73"$$

$$d_p = (8' - 9") - 1' - 0" = 7' - 9" = 93"$$

$$j_p = 93 - 11.78/2 = 87" = 7.26'$$

$$j_s = 105 - 8 - 11.78/2 = 91.11" = 7.6'$$

Example of a Pier Cap Design

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$$f_{pu} = f_{pe} + 900 \left(\frac{d_p - y_u}{I_e} \right) \leq f_{py}$$

$$= 0.6 \times 270 + 900 \left(\frac{93 - 14.73}{43 \times 12} \right)$$

$$= 162 + 136 = 298.5 \text{ ksi} > f_{py}$$

$$\therefore f_{pu} = f_{py} = 270 \times 0.9 = \underline{243 \text{ ksi}}$$

$$M_u \leq \phi M_n$$

$$\leq 0.9 (A_p \cdot f_{pu} \cdot j_p + A_s \cdot f_y \cdot j_s)$$

$$0.9 [(57 \times 0.217 \times 243 \times 7.26) + (A_s \times 60 \times 7.6)]$$

$$19,639 + 410.4 \times A_s = 24,165$$

$$410.4 A_s = 4526.00$$

$$A_s = \frac{4526.00}{410.4} = 11.00 \text{ in}^2$$

$$\text{Select \#8 bar} \rightarrow A_s \text{ per bar} = 0.79 \text{ in}^2$$

$$\text{Number of \#8 bar required: } 11/0.79 = 14$$

$$\text{Provided: } 20 \text{ \#8 bars}$$

$$\Sigma A_s \text{ provided} = 20 \times 0.79 = 15.8 \text{ in}^2$$

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Refined calculations.

$$C = A_p \cdot f_{pu} + A_s \cdot f_y$$

$$= 57 \times 0.217 \times 243 + 15.8 \times 60$$

$$= 3005.67 + 948.00$$

$$= 3953.67 \text{ Kips}$$

$$C = 0.85 f_c' \times a \times 60 = 3953.67 \cdot 10^3$$

$$a = \frac{3953.67 \cdot 10^3}{0.85 \times 5000 \times 60}$$

$$= 15.5"$$

$$j_p = 105" - 12 - 15.5/2 = 85.25"$$

$$= 7.10'$$

$$j_s = 105" - 8 - 15.5/2 = 89.25"$$

$$= 7.44'$$

$$M_n = 3005.67 \times 7.1 + 948 \times 7.44$$

$$= 28,393.37 \text{ K-FT}$$

$$\phi M_n = -0.9 (28,393.37)$$

$$= -25,554 \text{ K-FT}$$

$$> | -24,165 \text{ K-FT} | \quad \text{O.K.}$$

$$PPR = \frac{A_p \cdot f_{py}}{A_p \cdot f_{py} + A_s \cdot f_y}$$

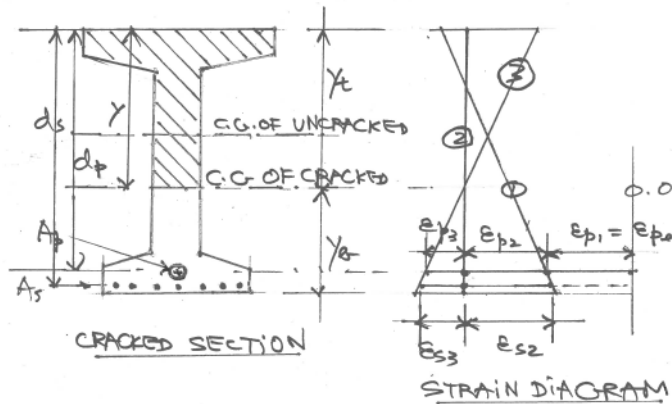
$$= \frac{3005.67}{3953.67}$$

$$= \underline{0.76}$$

Example of a Pier Cap Design

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CRACKED SECTION ANALYSIS (GENERAL)

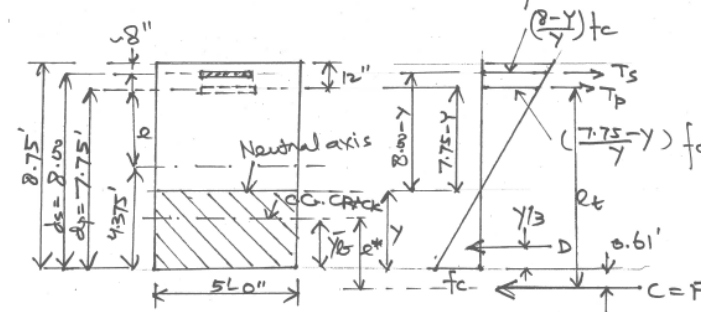


LOADING STAGES

- ① Pref. alone. (FULL SECTION)
- ② DECOMPRESSION (FULL SECTION)
- ③ Pref. PLUS TOTAL LOADS AT SERVICE (CRACKED SECTION)

FDOT	FPID No.	Sheet No. 16 of
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CRACKED SECTION ANALYSIS



$$A_p = 3 \times 19 \times 0.217 = 12.37 \text{ in}^2$$

$$A_s = 20 \times 0.79 = 15.8 \text{ in}^2$$

$$P_{ef} = 57 \times 58.6 \times 0.6 = 2004.12 \text{ Kips}$$

$$i^2 = \frac{I_c}{A_c} = \frac{279.13}{43.75} = 6.38 \text{ ft}^2$$

$$f_{p1} = f_{p0} = \frac{P_{ef}}{A_p} = \frac{2004.12}{12.37} = 162 \text{ ksi}$$

$$f_{tp} = 28,500 \text{ ksi}$$

$$f_{ts} = 29,000 \text{ ksi}$$


$$f_{tc} = W_c^{1.5} \cdot 33 \sqrt{f'_c} \quad W_c = 145 \text{ PCF}$$

$$= 145^{1.5} \cdot 33 \cdot \sqrt{5000}$$

$$= 4074.28 \text{ ksi}$$

$$= 586.696 \text{ KSF}$$

Example of a Pier Cap Design

 Structures Design Office	FPID No.		Sheet No. 17 of
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Compute Modular Ratio

$$m_p = E_p / E_c = 28,500 / 4074.28 = 7.0$$

$$m_s = E_s / E_c = 29,000 / 4074.28 = 7.12$$

$$\epsilon_{p2} = \frac{P_2}{A_c \cdot E_c} \left(1 + \frac{e^2}{i^2} \right)$$

$$= \frac{2004.12}{43.75 \times 586.696} \left(1 + \frac{3.375^2}{6.38} \right)$$

$$= 0.000217$$

$$f_{p2} = 0.000217 \times 28,500$$

$$= 6.19 \text{ KSI.}$$

$$C = F = A_p (f_{p1} + f_{p2})$$


$$= 12.37 (162 + 6.19)$$

$$= 2080.6 \text{ Kips.}$$

$$M_t = M_{DL} + M_{(LL+I)}$$

$$= 17,393.41 \text{ K-FT.}$$

$$e_t = \frac{M_t}{F} = \frac{17,393.41}{2080.6} = 8.36'$$

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$$T_s = \frac{(8.00 - \gamma)}{\gamma} \times f_c \times A_s \cdot m_s$$

$$= \frac{(8.00 - \gamma)}{\gamma} \times f_c \times \left(\frac{15.8}{144} \right) \times 7.12$$

$$= \frac{(8.00 - \gamma)}{\gamma} \times f_c \times 0.781 \text{ Kips}$$

$$T_p = \frac{(7.75 - \gamma)}{\gamma} \times f_c \times \left(\frac{12.37}{144} \right) \times 7.0$$

$$= \frac{(7.75 - \gamma)}{\gamma} \times f_c \times 0.601 \text{ Kips.}$$

$$D = \frac{1}{2} \times f_c \times \gamma \times 5' = 2.5 f_c \cdot \gamma$$

$$\Sigma M = 0 \text{ (about C)}$$

$$\therefore T_s \times 8.61 + T_p \times 8.36 = D (\gamma/3 + 0.61)$$

$$\left[\frac{(8.00 - \gamma)}{\gamma} \times 6.72 f_c \right] + \left[\frac{(7.75 - \gamma)}{\gamma} \times 5.024 f_c \right] = 2.5 f_c \cdot \gamma (\gamma/3 + 0.61)$$

$$\left[\frac{(8.00 - \gamma)}{\gamma} \times 6.72 \right] + \left[\frac{(7.75 - \gamma)}{\gamma} \times 5.0 \right] = 2.5 \gamma (\gamma/3 + 0.61)$$

$$\gamma \approx 3.4'$$

Example of a Pier Cap Design

FDOT Structures Design Office	FPID No.	Sheet No. 19 of
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CRACKED SECTION PROPERTIES

SECTION	A	y	AY	AC(y-y) ²	I _o
3.4x5	17	1.7	28.9	1.295	16.376
As. Ms.	0.781	8.00	6.24	36.288	0.0
Act =	17.781		35.14	37.583	16.376

$$\bar{y}_b = 35.14 / 17.781 = 1.976'$$

$$\bar{y}_t = 8 - 1.976 = 6.023'$$

$$I_t = 37.583 + 16.376 = 53.959 \text{ FT}^4$$

Ignore A_p mp since the PT is unbonded.

Check stresses of cracked section

$$\begin{aligned} f_{cs} &= -\frac{C}{A_{ct}} - \frac{C \cdot e^* \cdot \bar{y}_b^*}{I_t} \\ &= -\frac{2080.6}{17.781} - \frac{2080.6 \times (1.976 + 0.61) \times 1.976}{53.959} \\ &= -117.01 - 197.033 \\ &= -314.04 \text{ KSF} \\ &= |-2180.85 \text{ PSI}| > |1 - 0.4 f'_c| \\ &= |1 - 2000 \text{ PSI}| \\ \text{Use : } f'_c &= 6000 \text{ PSI.} \end{aligned}$$

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Steel Stress.

$$\begin{aligned} f_{s3} &= m_s \left[-\frac{C}{A_{ct}} + \frac{C \cdot e^* (d_s - \bar{y}_b^*)}{I_t} \right] \\ &= 7.12 \left[-\frac{2080.6}{17.781} + \frac{2080.6 \times 2.586 (8 - 1.976)}{53.959} \right] \\ &= 7.12 [-117.01 + 600.67] \\ &= +3443.66 \text{ KSF} \\ &= +23,914 \text{ PSI} \\ &= +23.914 \text{ KSI} = 165 \text{ MPA} \end{aligned}$$

COMPUTE CRACK WIDTH


① CEB-FIP 1970

$$w = (f_s - 40) \cdot 10^{-3} \text{ mm}$$

f_s = steel stress in MPA.

$$\begin{aligned} w &= (165 - 40) \cdot 10^{-3} \\ &= 0.125 \text{ mm} \\ &= 0.0049'' \end{aligned}$$

Example of a Pier Cap Design

 Structures Design Office	FPID No.		Sheet No. 21 of
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② GERGELY - LUTZ (1968)

$$w = 0.076 R f_s \sqrt[3]{d_c \cdot A_s} \cdot 10^{-3} \text{ (in.)}$$

where:

w = crack width.

R = Ratio of distance from tension face and steel centroid to neutral axis.

f_s = Tensile stress in reinforcing steel after decompression.

d_c = Concrete cover to center of closest bar layer

A_s = concrete tensile area per bar

$$w = 0.076 (1.17) (23.914) \sqrt[3]{(4)(46.5)} \cdot 10^{-3}$$

$$= 0.0121''$$


Add 6 # 8 bars (2ND row)

$$w = 0.076 (1.168) (23.914) \sqrt[3]{(4)(34.9)} \cdot 10^{-3}$$

$$= 0.0110''$$

Adding more rebars will not impact crack width significantly.

TO reduce crack width, adding PT will have significant impact.

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③ AASHTO LFD

$$f_s = \frac{Z}{(d_c \cdot A_s)^{1/3}} \leq 0.6 f_y$$

where:

Z = Quantity limiting spacing / distribution of flexural reinforcement. (kips/in)

$$Z = f_s \cdot (d_c \cdot A_s)^{1/3}$$

$$= 23.914 (4 \times 46.5)^{1/3}$$

$$= 136.5 \text{ K/in} > 130 \text{ Kips/in}$$


(severe exposure)

$$< 170 \text{ Kips/in}$$

(Moderate exposure)

$$f_s = 23.914 \text{ ksi} < 0.6 \times 60 = 36 \text{ ksi}$$

Example of a Pier Cap Design

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④ CP-110

$$f_{s3} \text{ at service} = 23.914 \text{ ksi} \\ = 165 \text{ MPA.}$$

Uncracked concrete stresses at service:

$$f_t = +839 \text{ psi} \\ = +5.78 \text{ MPA.}$$

$$\text{Concrete strength: } f'_c = 5000 \text{ psi} \\ = 34.48 \text{ MPA}$$


$$\text{Section depth} = 8'-9" \\ = 2667 \text{ mm}$$

$$\therefore \text{depth factor} = 0.7$$

$$\text{Allowable concrete tensile stress:} \\ 5.78 \times 0.7 = 4.05 \text{ MPA.}$$

\therefore concrete strength must be revised to 40 MPA (5800 psi)

Per Table 6.1, for grouted PT tendons, crack width of 0.1 mm (0.004") is obtained.

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CRACKED WIDTH CALC. SUMMARY

$$① \text{ CEB-FIP 1970: } 0.005"$$

$$② \text{ GERGELY-LUTZ: } 0.011"$$

$$④ \text{ CP-110: } 0.004"$$

$$(f'_c = 6000 \text{ psi}) \\ \text{AVERAGE: } 0.0067"$$

PER FDOT 400-21 (TABLE 1)
The proposed structure is acceptable for SLIGHTLY AGGRESSIVE ENVIRONMENT.

Presentation Outline

1. Background
2. Introduction to Partial Prestressing
3. Design Approach
4. Example of a Pier Cap Design
5. Potential applications

Potential Application

- Cast-in-place structures in general
- Pier Cap
- Straddle beam
- Footing
- Pier column
- Transverse Design for box girder
- Precast girders with large camber
- Deck slab
- Arch bridge

Thank you for your attention
Any questions?

